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Paul Girard Blake

ECONOMY OF COMPOSITE STRINGERS

FOR

SHORT SPAN HIGHWAY BRIDGES

A THESIS

Presented to

the Faculty of the Graduate Division

by

Paul Girard Blake

In Partial Fulfillment

of the Requirements for the Degree

Master of Science in Civil Engineering

Georgia Institute of Technology

June, 1957

ECONOMY OF COMPOSITE STRINGERS

FOR

SHORT SPAN HIGHWAY BRIDGES

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Date Approved by Chairman: May 31, 1957

ACKNOWLEDGEMENTS

The writer is hereby indebted to the Automotive Safety Foundation for granting him a fellowship to study at the Georgia Institute of Technology during the time in which this thesis was prepared.

To the entire staff of the Civil Engineering Department, with particular emphasis on my Faculty Advisor, Professor Radnor J. Paquette, and Dr. F. W. Schutz, Jr., and Professor Austin B. Caseman of the committee are expressed thanks for guidance, patience and friendship. In addition, the Lincoln Arc Welding Foundation supplied literature and encouragement.

To the writer's entire family and to Professor Littleton of Tufts University, he owes much for continued encouragement and confidence during both the dark and bright periods of this year of study.

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ABSTRACT

ECONOMY OF COMPOSITE STRINGERS

FOR

SHORT SPAN HIGHWAY BRIDGES

Paul Girard Blake

55 Pages

Directed By Professor Radnor J. Paquette

This study was undertaken to determine the relative economic positions of composite wide flange beams and composite welded plate girders in the field of short simple span, deck type highway bridges. In determining this, the length of span, stringer spacing, class of live load, and the condition of beam support during the construction of the concrete slab were investigated as to their effect.

During the study a quick method of computing areas of trial sections for composite welded plate girders was developed.

The problem was approached by designing interior stringers of both types, for spans of fifty to one hundred feet, using a six and eight foot spacing. The live load was considered for two classes, the AASHO H20 S16 and H15 S12. Beams were designed for no intermediate support during construction of the concrete deck, and for the case of full support during construction of the deck. The economic comparison was made by the weight of steel required to support the dead and live load.

Accepted design office practice was used throughout the study, and the AASHO bridge specifications followed.

Under all conditions studied, the composite welded plate girder had favorable economic weight saving characteristics. This was particularly true for the 100 foot spans, for all live load and spacing combinations studied, where the weight savings were near fifty per cent. For all spans the weight savings were large enough to warrant investigation in relation to other design considerations.

The economy of welded girder is due to three factors:

(1) Use of increased web depth.

(2) Location of cover plate areas in the most advantageous position.

(3) Reduction of dead load moments due to the use of lighter members.

Supporting steel members during construction of the concrete deck indicated too small weight reductions to justify the added construction cost.

The steel weight saved by using the H15 S12 live load was not sufficient to sacrifice the lost live load capacity.

Varying stringer spacing from six to eight feet gave inconclusive results with the wide flange members, but indicated a small saving with the welded girder. The square foot dead load weight was fifteen per cent greater for the eight foot spacing.

Because of the larger moment of inertia, for the same span length, the welded girder had better resistance to live load deflections.

Bridge design engineers should investigate the potential economic value of the composite welded plate girder spans, both for the saving of material and design time, when making economic studies for bridges of this type.

CHAPTER I

INTRODUCTION

The impact of the automobile on the American scene is expressing itself in many ways. Among the most important of these is the system of limited access highways that are threading their way over the country. Safety and high speed requirements of these arteries of transportation have led to the elimination of at-grade intersections by using bridge structures. As a result more bridges are being constructed and more are being anticipated per road mile than ever before. An example of this is the 500 bridges on the New York State Thruway (1).

A class of highway bridges has developed that can no longer be included among the architectural edifices required to cross natural obstacles. These bridges are the short span grade separators that must, by nature, be simple, functional, and economical. The materials, labor, and time used in the construction of a bridge increase the cost of a road greatly. Careful economic studies of the various elements of a proposed structure should be made.

Since the early work of Westergaard on bridge slabs, and the development of improved shear connectors, bridge engineers have become increasingly dependent on the use of composite design to develop economy (2). The use of this type of design permits a reduction in the size of the steel section, resulting in less dead weight. Bridges often become geometric control features for highway alignment design as they partially control the underpass clearance requirements. In relation to bridges of the older type that carried the total dead and live load on the steel section, the composite concrete and steel beam bridge reflected an additional saving in embankment fill.

In more recent years, the use of welded plate girders has become increasingly popular. In a survey made by the Lincoln Arc Welding Foundation in November 1956, 22 states reported using all welded plate girders, compared to 10 states in 1950 (3). With welding now a part of the everyday life of the bridge engineer, the question arises as to the economic application of composite concrete and welded steel plate girders to deck type bridges.

The advantages of composite design are in the utilization of a portion of the concrete deck to resist flexure stresses in compression. These stresses may be from dead or live loads. The use of a portion of the concrete deck develops greater resistance to bending without increasing the weight of steel used. The wide flange sections, because of their equal size flanges and thick webs, do not make the best use of the steel area available. By using plates welded in the shape of an I, the steel can be located to greater advantage. The webs can be made thinner and deeper, resulting in a section of higher moment of inertia and less weight. These considerations led to this study of a comparison of composite concrete and steel highway bridge spans using rolled wide flange sections and those made of stiffened welded plate girders.

In most types of economic comparisons, several factors influence the results. Some of these, because their beneficial and adverse effects balance, can be eliminated. Others, such as dollar costs, have in themselves many dependent factors that would not present a clear picture. A common basis of weight of steel was selected to show relative economy. This mode of comparison lacks some elements needed to show the overall picture, but it does show whether or not sufficient material is saved by using welded plate girders to warrant further investigation.

A method of study that would show the relative effects of span lengths, stringer spacing, and different types of live loading is used. The cases of stringers supported and unsupported during the placing and curing application of the concrete deck slab are compared. Each case is treated as it would be by a bridge design engineer, using accepted practice as is specified by the American Association of State Highway Officials (4). Two limitations are placed on the study. The first limits the work to interior stringers. The exterior or facia stringer of a bridge is dependent on the type of parapet used, and the location of the stringer to the traveled part of the deck. The second limitation is that the comparable weights on steel sections selected are based on the maximum moment of the design span. Further savings can be made by reducing cover plates near supports. This would reflect greater savings for the welded sections so that the comparison of weight at the point of maximum moment is conservative.

The cost of welding the plates is not considered, as this is a phase of investigation that would follow the establishment of sufficient saving of steel weight.

This study is made in an effort to determine within the limitations placed on it, if the composite concrete and welded plate girder type of design has a place in the field of short span highway bridges.

CHAPTER II

THEORY

Composite beams are proportioned by the moment of inertia method, following the elastic theory. The following assumptions are made in designing for elastic conditions:

(1) A plane section before bending is a plane section after bending, and remains normal to the longitudinal fibers.

(2) Stress is proportional to strain.

(3) Strains are proportional to the distance from the neutral axis.

(4) The neutral axis coincides with the centroid of the section.

(5) Stresses are algebraically additive.

The stresses in the steel and concrete are proportional to their moduli of elasticity.

The moment of inertia of a plane section is equal to the sum of the moments of inertia of its parts about their own centroids, plus the product of the areas times the square of the distance from their centroids to the centroid of the whole.

In Figure 1, the stresses in the various parts of the section are computed by the following equations:

$$f_{st}\ (DL) = \frac{M_{DL}\ (d - C_1)}{I\ \text{non composite}}$$

$$f_{st}(LL + I) = \frac{M_{(LL + I)} (d + t - C_2)}{I \text{ composite}}$$

$f_{st}(DL) + f_{st}(LL + I)$ is less than f_s allowable

$$f_{sc}(DL) = \frac{M_{(DL)} C_1}{I \text{ composite}}$$

$$f_{sc}(LL + I) = \frac{M_{(LL + I)} (C_2 - t)}{I \text{ composite}}$$

$f_{sc}(DL) + f_{sc}(LL + I)$ is less than f_s allowable

$$f_c = \frac{M_{(LL) + I} (C_2)}{E_s / E_c \times I \text{ composite}}$$

f_c is less than f_c allowable

where

$f_{st}(DL)$ is the tensile steel stress from dead load

$f_{st}(LL + I)$ is the tensile stress from live and impact load

$f_{sc}(DL)$ is the compressive steel stress from dead load

$f_{sc}(LL + I)$ is the compressive steel stress from live and impact load

f_c is the compressive stress in concrete

$M_{(DL)}$ is the dead load moment

$M_{(LL + I)}$ is the live and impact moment

I is the moment of inertia

Other symbols are shown on the figure.

The moment of inertia of the composite section is equal to:

$$I_{non\ composite} + A_s (C_1 - C_2 + t)^2 + \frac{A_c}{E_{s/D_c}} (C_2 - t_2)^2$$

This neglects the moment of inertia of the slab about its own axis, a relatively small quantity.

In the design of sections supported during the placing and curing of the concrete slab, consideration is given to the effect of creep in the concrete under sustained loads by increasing $E_{s/E_c} = n$, to 3n.

Buckling of thin webs is prevented by the use of stiffener plates.

The transfer of shear between the concrete and steel section is accomplished by using shear devices in the form of lugs, bulb angles, or spiral reinforcing rods welded to the top flange of the steel section. The devices transfer horizontal shear as measured by $q = \frac{VQ}{I}$

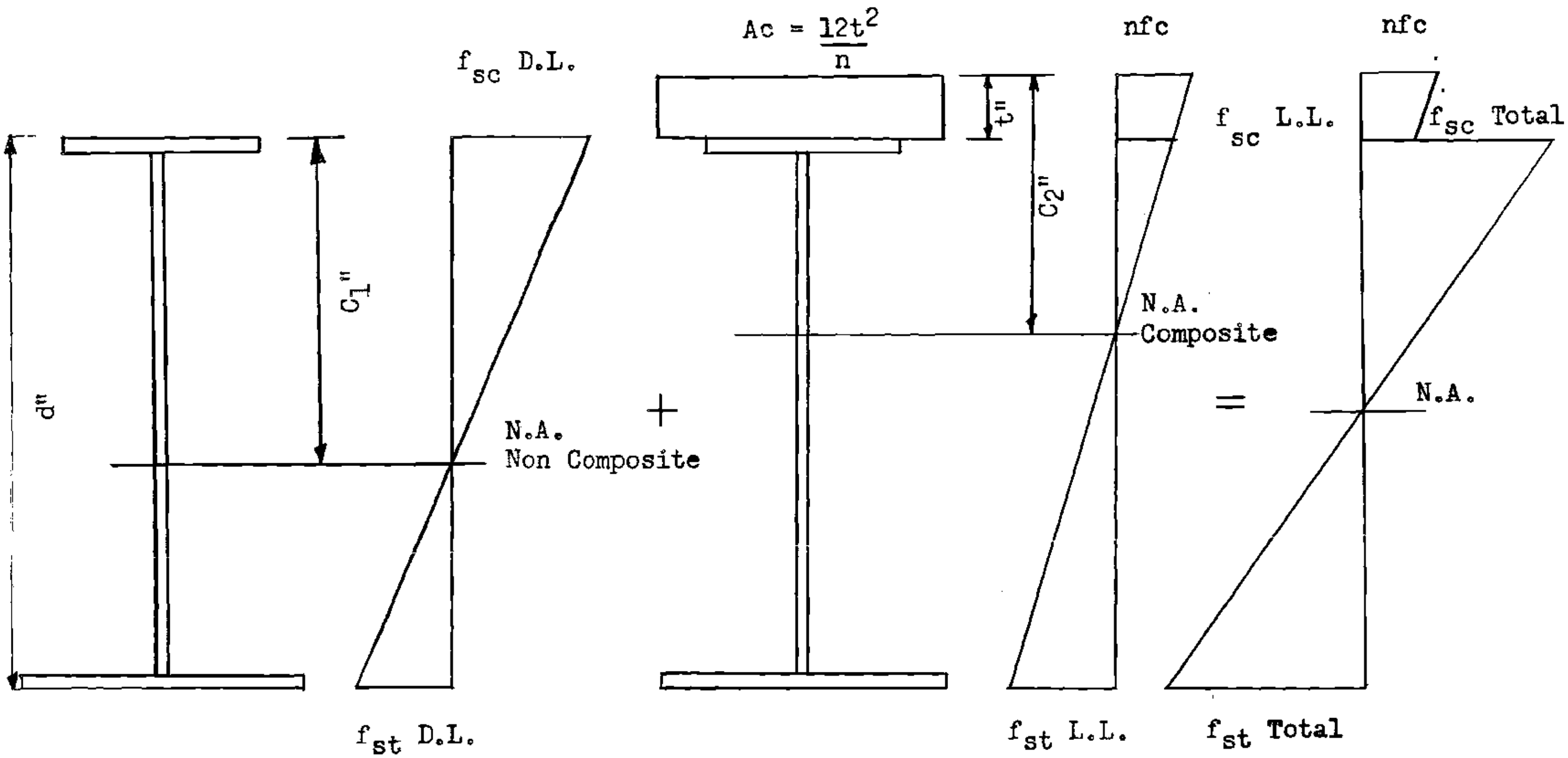
where q is the unit shear flow in pounds per inch

V is the total shear in pounds

Q is the computed statical moment of the area above the plane being investigated

I is the moment of inertia of the section (5)

This same equation is used to compute weld stresses for cover plate connections.



Steel Section
Stress from Dead Load

Composite Section
Stress from Live Load

Combined Stress Diagram

Fig. 1. Dead Load and Live Load Stress Diagram

CHAPTER III

DESIGN PROCEDURE

The design of the vsrious parts of the composite beams being studied is governed by the American Association of State Highway Officials Standard Specifications for Highway Bridges, Sixth Edition, 1953.

Highway live load.--Two tractor trucks with semi trailer loadings, the H20 S16 44 and the H15 S12 44, were selected for comparison (6). The H20 S16 is the standard loading recommended by the AASHO for designing primary highway structures. An H15 S12 loading is the minimum permitted for highways subjected to heavy truck traffic. The comparison of the two loadings is to determine if the saving of steel is appreciable by designing for the lighter load. Figure 2 shows the application of the loads to determine the maximum moment.

The distribution of the wheel loads for interior stringers of a structure carrying two or more traffic lanes is:

$$P = WL \times (1 + I) \times \frac{S}{5}$$

where P is the load on the stringer

WL is the wheel load

I is the impact factor

S is the average spacing of the stringers (7) now $\qquad I = \qquad L = \dfrac{50}{125}$, with a maximum $I = 0.30$

where $\qquad L =$ is the portion of the span loaded to produce maximum stresses (8).

<u>Allowable Stresses.</u>--Steel, tension in extreme fiber ----- 18,000 psi

compression in extreme fiber of	
Continuously supported flanges -------	18,000 psi
axial compression of stiffeners -----	18,000 psi
Concrete, compression of extreme	
fiber ------	1,200 psi
Ratio $E_s/E_c = n$, for live load	10
for dead load	30 (9).

<u>Stringer spacing.</u>--The spacing of the stringers depends on the economic use of the concrete deck. Good practice limits the depth of the structural slab to the minimum required for proper placing and cover of the reinforcing steel. A second consideration in the slab selection is the limitation placed on the portion of the slab that may be used for composite design. This limits the area to the least of twelve times the thickness of the slab, or to the center to center distance between beams (10).

To show the relative effect of spacing, a case of six and eight foot spacing is used. The six foot spacing requires a structural slab of six inches, and an added wearing surface of one inch. The wearing surface is considered for weight, but is assumed to have no structural value. The eight foot spacing requires a seven inch structural slab and a one inch wearing surface. By using the six foot spacing, the rule of twelve times the thickness of slab applies. The eight foot spacing is governed by the center to center of beams.

Since the stringer spacing governs the distribution of live load, this gives a comparison of medium and heavy span loadings.

Slabs were designed for reinforcing placed perpendicular to the direction of traffic (11). In the selection of the thickness of slabs for both spacings, the concrete design was nearly balanced. An additional thickness was then added to facilitate the placing of reinforcing.

The effective areas of concrete for composite design are:

Six foot spacing	$n = 10$,	43.3 square inches
	$n = 30$,	14.4 square inches
Eight foot spacing	$n = 10$,	58.8 square inches
	$n = 30$,	19.6 square inches

Spans.--The upper limit of span length was fixed at one hundred feet, as this represents the economic limit for using composite rolled wide flange beams with plates at the design spacing. As it is preferred to keep the neutral axis below the concrete slab, a lower limit of span length of fifty feet was selected (12). Below this point, the ratio of concrete area to steel area would put the neutral axis in, or near the concrete slab. Intermediate spans of 62.5, 75, and 87.5 feet were designed to determine possible curve trends.

General design assumptions.--As a result of the assumptions made and the safety factors involved, it was decided to make all computations to slide rule or three significant figure accuracy. This also would be the procedure used in a practical design.

Since the design procedure is by trial and error, the allowable tolerance of 18,000 $\pm$ 500 psi was permitted in checking steel stresses. Concrete stresses are generally not critical in interior concrete stringers.

A variation of twenty five pounds per linear foot was permitted in the design dead load. The small variation caused by this decreased with increased span length.

Computation of moments.--Moments were computed for a uniform dead load that included the weights of the slab and wearing surface, the steel, and an allowance for diaphragms and stiffeners when applicable.

The live load moments were computed by applying the distribution and impact factors to the tables of moments for H20 S16 and H15 S12 live loadings in the AASHO bridge specifications (13).

Design of composite wide flange beams.--The method used to design these sections was by trial and error. The total moment and the ratio of dead load moment to the total moment were selected from Table 1 for the desired span length. By interpolation on Figs. 5, 6, and 7, the trial wide flange section and cover plate were determined. The non-composite and the composite moments of inertia were computed and checked. If the combined fiber stress in tension was within allowed tolerance and the compression fiber stress showed reasonable economy, the section was considered satisfactory for use in the comparison. Seventy five per cent of the allowable stress in compression was felt to be a reasonable figure. When necessary, the trial sections were adjusted by adding or removing cover plate area.

In no case was the concrete stress critical.

Design of composite welded plate girders.--The moments used to design these sections are listed in Table 2.

The selection of the trial section was done in two parts. A balanced I section was designed to carry dead load only, using the web to resist moment. A 3/8 inch web plate was used in all cases. This is the minimum size web used for exposed steel, and permits the use of web depths up to 63 inches with transverse stiffeners.

The live load moment was considered to be resisted by a couple, formed by stresses on the effective concrete area and an area of steel, stressed to the allowable, and having the same horizontal axis as the bottom plate of the I section. In computing these areas, the depth of the web plate was considered by trying six inch increments, and checking the savings in steel. If the savings in steel were less than two square inches, or if the top plate size was too small to weld shear connectors onto, the shallower section was designed. All cover plates were assumed to be one inch thick.

The non-composite moment of inertia is based on the total steel area of the I section and the steel needed for live load moment.

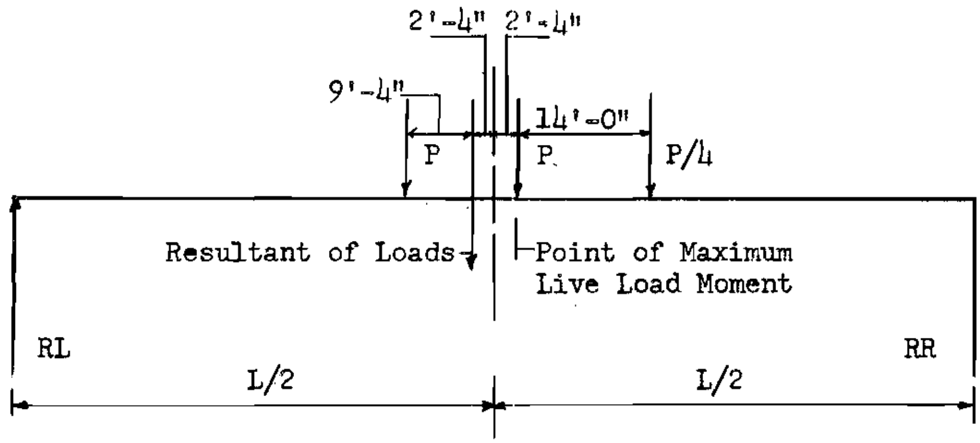
The composite moment of inertia was computed in the usual manner. When the trial section did not satisfy the stress tolerance, the cover plate areas were adjusted and new moment of inertia computed. The stresses were then checked using the new moment of inertia. This adjustment was necessary only with the fifty foot spans. This procedure of trial design is shown schematically on Figure 4.

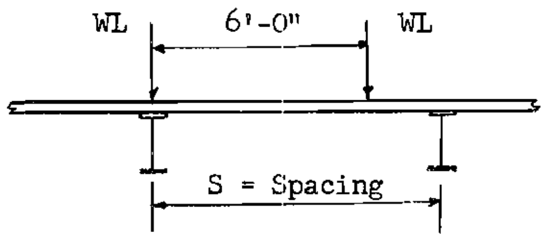
Stiffener plates were designed for the maximum shear at the maximum spacing permitted by the code (14). The design load shears were less than allowed, in the region investigated. A closer spacing than maximum was needed only near the supports. Table 11 shows the weight of stiffener on a per foot basis.

Design of composite beams supported for dead load.--Both the wide flange and the welded plate sections were treated in the same manner for this comparison. The section resisting dead load was recomputed using an additional area of concrete in the ratio of $n = 30$. The new stresses were determined, and necessary adjustments made to the cover plate areas to meet the tolerance of stress. It was found early in the study that the saving in steel was small, for all spans, and spans of 50, 75, and 100 feet were checked.

It was found that, theoretically, the top plate of the welded girder could be eliminated for 50 foot spans. This would not permit proper shear transfer, and some plate was used to maintain a practical design.

Design of non-composite sections.--A series of beams were designed for the total moment being resisted by a wide flange section to show the weight saving by using composite beams. The moments in Table 1 were used. These weights are shown on Figure 8.





H20 S16, WL = 16 kips
H15 S12, WL = 12 kips

$$P = WL \times \frac{S}{5} \times \left(1 + \frac{50}{L + 125}\right)$$

Fig. 2. Maximum Live Load Conditions

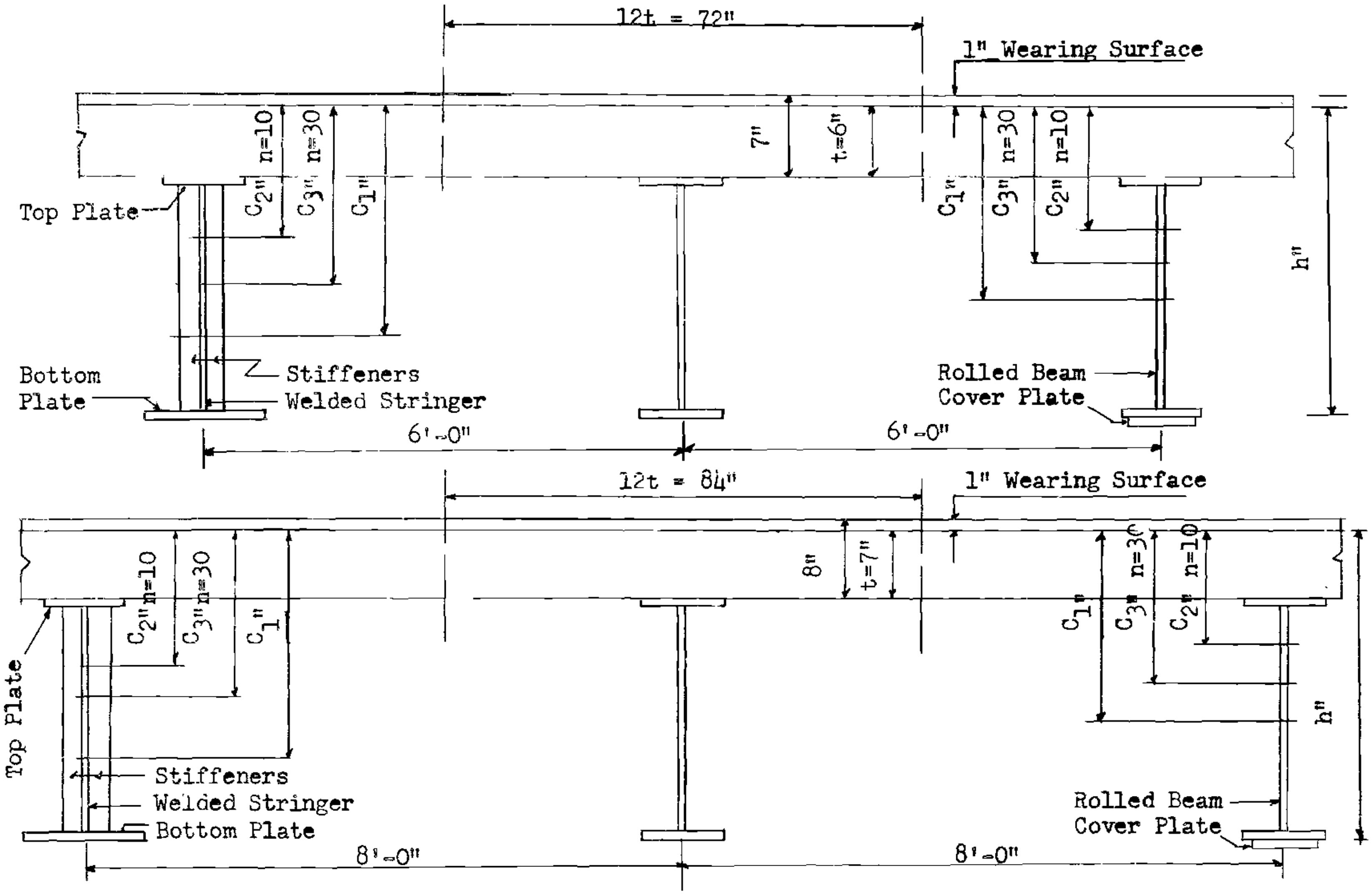
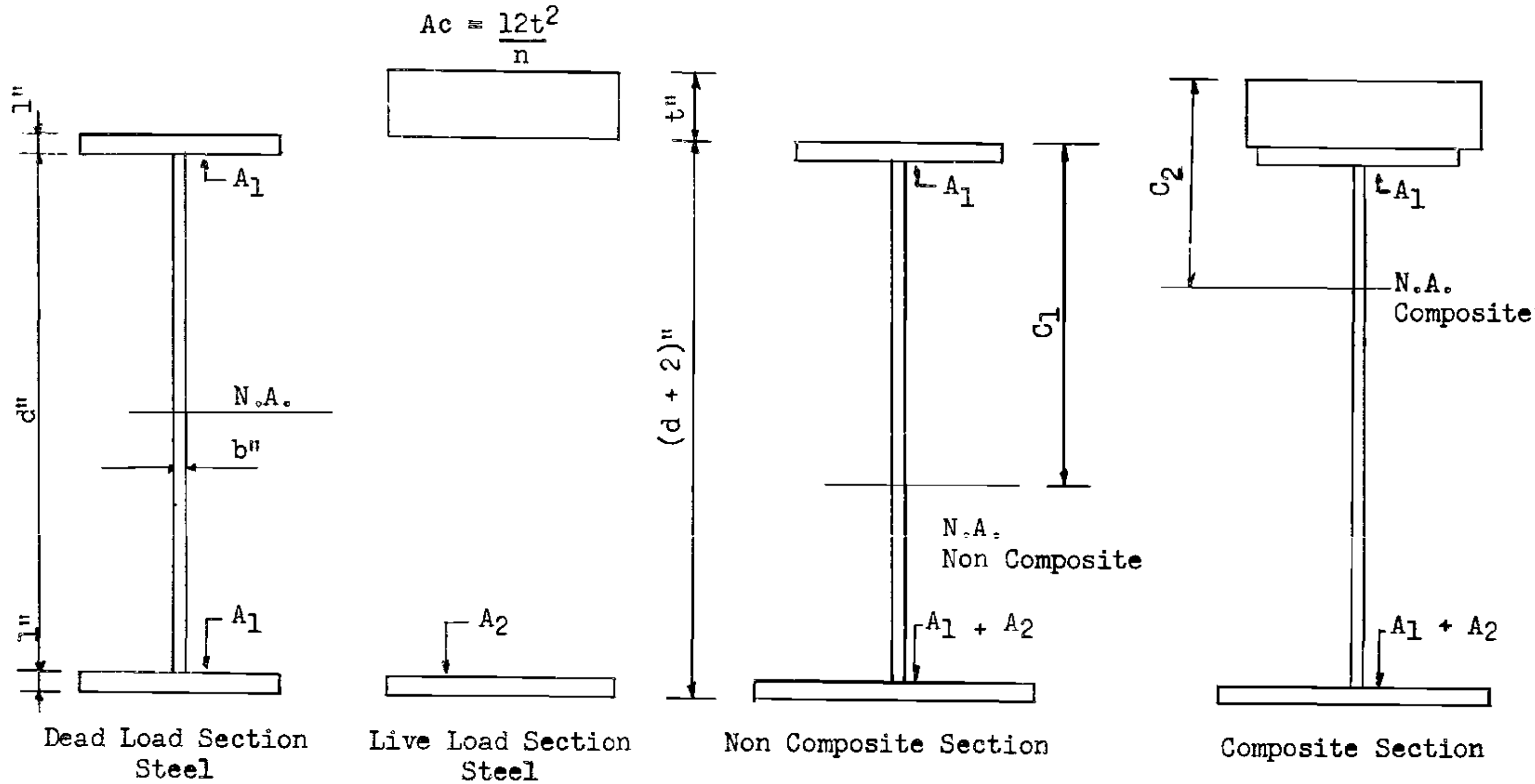


Fig. 3. Typical Bridge Deck Cross Sections



$$A_1 = \frac{M_{DL}\ (d + 2)}{f_s\ \text{all.}\ (d + 1)^2} - \frac{bd^3}{6(d + 1)^2}\ ; \quad A_2 = \frac{M_{LL}}{f_s\ \text{all.}\ d + \frac{(3+t)}{2}}$$

Fig. 4. Approximate Method of Selecting Composite Welded Plate Areas

CHAPTER IV

DISCUSSION OF RESULTS

The properties and resulting stresses for the composite sections designed are shown in Tables 3 to 10.

Comparison of composite wide flange beams to composite welded plate girders.--The composite welded plate girders showed a considerable weight saving advantage over the composite wide flange beams under all conditions studied. Figures 11 and 12 illustrate how the weight saving increases with an increase in span length. These curves are plotted using a ratio of welded girder weight to wide flange beam weight for each span studied.

Using the H20 S16 live loading and six foot spacing, the savings ranged from 19 per cent or 22 pounds of steel, for a fifty foot span, to 47 per cent or 149 pounds for a one hundred foot span. The members designed for an H20 S16 live loading and an eight foot spacing resulted in a 26 per cent saving that is equivalent to 40 pounds of steel for a fifty foot span. The one hundred foot span welded girder was 195 pounds or 48 per cent lighter.

The H15 S12 live loading showed comparable results. For a six foot spacing, the fifty foot span showed a 24 pound or 20 per cent saving and the one hundred foot span showed a 130 pound or 43 per cent steel weight advantage. Using an eight foot stringer spacing, the fifty foot span had a 30 pound, 22 per cent saving; and the one hundred foot span had a 143 pound or 47 per cent saving.

The steel weight ratio curves do not have a uniform slope, but in general the characteristics of the curves are similar. For the shorter spans of 50 and 62.5 feet, the percentage increase in steel savings was not rapid; however the increase is greater as the span increases. There is some tendency for the curve to flatten out near the one hundred foot span. This could indicate either that the limit is being reached for a sixty inch web in the welded girder or that the larger weights of steel in the longer spans make percentage changes smaller, even though the pound differential is still increasing.

To gain some of the advantage of using deeper sections in the welded plate girder additional expenditures for concrete and earth embankments are required. The unit prices of these materials are relatively low enough to justify the use of large quantities of them to save steel.

<u>The affect of live load</u>.--Reducing the live load from an H20 S16 to an H15 S12 does not result in large savings of steel. Figure 13 shows the relative percentages saved by using the lighter load. The ratios for the composite wide flange members do not give a definite trend, but they do indicate that the savings in steel are less than 13 per cent. For those two cases, an average ratio line is shown that indicates the general saving of steel weight is approximately ten per cent for a six foot spacing, and six per cent for the eight foot spacing. In terms of weight, the savings for the six foot spacing would be 10 pounds per foot for a fifty foot span and 24 pounds per foot for a hundred foot span. In the case of the eight foot spacing, the savings range from 18 to 61 pounds.

The welded plate girders also had an erratic ratio curve, but to a far smaller degree. Both spacings indicate that the saving of steel decreased with span length. This condition is expected as the dead load moment, carried by the steel section alone, becomes a larger part of the total moment as the span increases. The ratio of weight of steel required varied from 90 per cent for a fifty foot span, to 97 per cent for the hundred foot span, using a six spacing; and from 93 to 96 per cent using an eight foot spacing. In terms of weight saved this means a range of ten to fifteen pounds and eight to nine pounds respectively.

The small economy involved does not justify the large loss of live load capacity.

<u>The affect of stringer spacing.</u>--The composite wide flange beams did not show any consistent trend as to the relative economy of the two spacings studied. Figure 14 shows the variation of points for each span length. The reason for this variation may be in the fact that moment of inertia of wide flange sections does not increase uniformly with an increase in weight of section.

The welded composite girders show an advantage in using wider spacings. This advantage decreases in the middle range of spans, and is more apparent in short and long spans. For the welded members the weight savings ranged from one pound to ten pounds per foot of span for the H20 S16 loading, and from no savings to ten pounds with the H15 S12 loading.

In all cases the uniform dead load per square foot of the slab was nearly 15 per cent larger on the beams with an eight foot spacing.

Beams supported during application of dead load.—Supporting stringers while the concrete deck is applied resulted in little or no weight reduction in steel. Only two cases indicated a saving -- the H20 S16 and H15 S12 loadings on a hundred foot span with eight foot spacing. The steel weights saved were 34 and 10 pounds respectively. This amounted to an eight and three per cent reduction.

The effective area of concrete is reduced considerably to account for plastic flow and the full value of the concrete slab is therefore not realized. Stresses in the top flange of the wide flange members are reduced, but the section modulus for tensile stresses is not increased sufficiently to cause any reduction in steel area.

The welded plate girders used slightly less steel when supported during the construction of the slab. The top plate was reduced in size when possible and this accounted for most of the reduction.

The weight per foot of reductions are as follows:

	50' span	75' span	100' span
H20 S16, 6 foot spacing	10#	7#	14#
H20 S16, 8 foot spacing	13#	21#	19#
H15 S12, 6 foot spacing	3#	1#	9#
H15 S12, 8 foot spacing	5#	11#	19#

The small weights saved do not justify the additional cost of supporting the stringers during the construction of the concrete deck.

Relation of web depth to total moment for composite welded plate girders.--Figures 15 and 16 show how the depth of web increases with moment. The relationship is plotted as a straight line function. This is comparable to the straight line relationship of weight to span length of Fig. 10. The equations for these lines are:

$$\text{Six foot spacing,} \quad d = 36 + \frac{(M - 600)}{67},$$

$$\text{Eight foot spacing,} \quad d = 36 + \frac{(M - 750)}{75},$$

d is web depth in inches

M is total moment in foot kips.

By using the straight line relationship within the limits of the members designed, trial depth may be readily obtained. If the depth increases are limited to six inch increments the choice of depth will reduce to two possibilities.

Deflection characteristics.--The dead load deflection of a deck type stringer bridge is usually eliminated by cambering the stringers, so that the residual camper is larger than the dead load deflection.

In spans of the length considered, live load deflection is not usually a factor. However, it is desirable to have a minimum deflection. Fig. 17 shows the ratio of moment of inertia of composite welded plate girders compared to length. The graphs indicate that for the longer spans, around one hundred feet, the live load deflection of the welded plate girder would be only sixty per cent of that of the wide flange bridge.

In the shorter spans up to 87.5 feet, the ratio is nearly constant. This indicates that the depth of web is a controlling factor in this relationship. As the ratio of depth increases the ratio of moment of inertia also increases. Since the deflection is an inverse function of the moment of inertia, the ratios are plotted smaller over larger. The curve for the H20 S16 loading with an eight foot stringer spacing on Fig. 17 shows the effect of increasing depth on deflection. This case has an increase in depth ranging from 42 to 60 inches.

CHAPTER V

CONCLUSIONS

The composite welded plate girder has a definite weight saving advantage over the composite wide flange beam in deck type highway bridges. The welded girder develops this saving due to three factors:

(1) Use of increased web depth.

(2) Location of cover plate area in the most advantageous position.

(3) Reduced dead load moments resulting from lighter members.

Supporting steel members during the construction of the concrete deck does not materially reduce the steel area required.

For this type of bridge, the savings in steel weight by using the AASHO H15 S12 is not appreciable when compared to the load capacity forfeited.

The varying of stringer spacing from six to eight feet did not give conclusive results as to which spacing was most economical for composite wide flange beams.

For composite welded plate girders the eight foot stringer spacing was more economical, particularly when the fact that the eight foot spacing had a larger uniform per square foot dead load is considered.

In the design of this type of bridge deck, two factors must be considered. The first is the effective area of concrete permitted by the code, and the second is the dead load the steel must support. The dead load becomes an increasing part of the load the member has to carry as the span length increases, being about fifty per cent of the total in a 100 foot span. This load is carried on the steel section or on a composite section with a reduced concrete area.

The straight line relationship of depth to total moment as illustrated by Figures 15 and 16 will facilitate the rapid selection of trial depth in proportioning composite welded plate girders.

The composite welded plate girder has a larger moment of inertia than the composite wide flange beam for members carrying the same span. This will give relatively lower live load deflections when girder is used.

Building steel sections of welded plates is advantageous to the small fabricating mill, as stock piles of material would be more uniform and contain fewer items. Their use will permit more local fabricators to participate in highway building programs with a resulting time saving to the public.

CHAPTER VI

RECOMMENDATIONS

In cost comparisons for individual bridge sites, the advantages of composite welded plate girders should be recognized by bridge engineers as a possible money saving and time saving element.

Where bridges become geometric design controls, the weight savings offered by these sections should be checked with regard to increased embankment fill to prevent false economy in the use of shallow sections.

Further possibilities of study in this area is unlimited, both in research and design study. The development of thinner and lighter bridge slabs that could acceptably be combined with the composite theory would present considerable savings. A study of existing structures of this type to determine the cost ratio between superstructures and substructures would be advantageous.

APPENDIX

TABLES

Table 1. Design Moments for Composite Wide Flange Beams, Supported and Unsupported for Dead Load

Class of Live Load	Span Feet	Stringer Spacing Feet	Dead Load Moment Foot kips	Live & Impact Load Moment Foot kips	Total Moment Foot kips	Ratio of Dead Load to Total Moment
H20 S16	50.	6	207	485	692	0.299
"	67.5	6	329	647	976	0.340
"	75.	6	528	806	1334	0.396
"	87.5	6	718	965	1683	0.426
"	100.	6	1065	1110	2175	0.490
H20 S16	50.	8	297	645	942	0.315
"	67.5	8	487	862	1349	0.361
"	75.	8	773	1075	1848	0.418
"	87.5	8	1080	1285	2365	0.457
"	100.	8	1440	1480	2920	0.493
H15 S12	50.	6	203	363	566	0.359
"	62.5	6	330	485	815	0.405
"	75.	6	509	614	1123	0.453
"	87.5	6	742	723	1465	0.508
"	100.	6	1065	836	1901	0.561
H15 S12	50.	8	297	483	780	0.381
"	62.5	8	487	646	1133	0.429
"	75.	8	755	807	1562	0.483
"	87.5	8	1050	936	1986	0.529
"	100.	8	1410	1100	2520	0.555

Table 2. Design Moments for Composite Welded Plate Girders

Supported and Unsupported for Dead Load

Class of Live Load	Span Feet	Stringer Spacing Feet	Dead Load Moment Foot kips	Live & Impact Load Moment Foot kips	Total Moment Foot kips	Ratio of Dead Load to Total Moment
H20 S16	50.	6	195	485	680	0.287
"	62.5	6	307	647	954	0.322
"	75.	6	457	806	1263	0.362
"	87.5	6	663	965	1628	0.408
"	100.	6	908	1110	2018	0.450
H20 S16	50.	8	281	645	926	0.304
"	62.5	8	463	862	1325	0.349
"	75.	8	685	1075	1760	0.389
"	87.5	8	958	1285	2243	0.426
"	100.	8	1250	1480	2730	0.458
H15 S12	50.	6	195	363	558	0.349
"	62.5	6	305	485	790	0.386
"	75.	6	457	614	1071	0.426
"	87.5	6	655	723	1378	0.475
"	100.	6	875	836	1711	0.512
H15 S12	50.	8	281	483	764	0.368
"	62.5	8	452	646	1098	0.412
"	75.	8	667	807	1474	0.452
"	87.5	8	910	936	1846	0.493
"	100.	8	1220	1110	2330	0.524

Table 3. Design Properties and Maximum Stresses for Composite Wide Flange Beams, Unsupported for Dead Load, H20 S16 Live Load

Stringer Spacing 6' 0", n = 10, t = 6", 12t = 72"

Span Feet	h"	Wide Flange Section	Bottom Cover Plate	wt. lbs.	Non Composite C_1"	Non Composite I"4	Composite C_2"	Composite I"4	f_{st} ksi	f_{sc} ksi	f_c psi
50.0	36.3	30 W 108	6 x 1/2	118	16.3	5120	10.7	12,320	18.6	10.6	560
62.5	39.6	33 W 130	10 x 1/2	149	18.5	8140	13.8	18,400	18.0	12.3	585
75.0	42.8	36 W 150	10 x 1	184	21.3	11,780	16.3	26,400	18.4	15.4	618
87.5	43.5	36 W 194	12 x 1	235	21.5	15,530	18.1	31,780	18.3	16.3	658
100.0	43.5	36 W 280	12 x 1	321	20.6	22,500	19.2	39,250	17.9	16.1	656

Stringer Spacing 8' 0", n = 10, t = 7", 12t = 84"

Span Feet	h"	Wide Flange Section	Bottom Cover Plate	wt. lbs.	Non Composite C_1"	Non Composite I"4	Composite C_2"	Composite I"4	f_{st} ksi	f_{sc} ksi	f_c psi
50.0	42.8	36 W 150		150	17.9	9012	12.7	21,130	18.1	9.2	473
62.5	43.5	36 W 150	9 x 1	181	21.0	11,580	16.0	28,450	18.1	13.9	583
75.0	43.9	36 W 230	9 x 1	261	20.1	17,800	16.8	36,350	18.3	14.3	653
87.5	44.5	36 W 280	14 x 1	328	21.0	23,570	18.7	45,470	17.9	15.5	640
100.0	45.8	36 W 300	10 x 1 top 20 x 1 bottom	402	21.2	30,900	19.9	56,800	18.0	15.8	619

Table 4. Design Properties and Maximum Stresses for Composite Wide Flange Beams, Unsupported for Dead Load H15 S12 Live Load

Stringer Spacing 6' 0", n = 10, t = 6", 12t = 72"

Span Foot	h"	Wide Flange Section	Bottom Cover Plate	wt. lbs.	Non Composite C_1"	Non Composite I" 4	Composite C_2"	Composite I" 4	f_{st} ksi	f_{sc} ksi	f_c psi
50.0	35.8	30 W 108		108	14.9	4461	10.6	10,320	19.0	10.1	448
50.0	36.1	30 W 130		130	16.6	6699	12.2	13,100	15.0	8.2	404
62.5	36.8	30 W 108	7 x 1	132	17.8	6340	12.8	15,140	17.5	13.7	492
75.0	40.1	33 W 130	10 x 1	164	20.1	9000	14.8	21,200	18.3	16.6	523
87.5	43.5	36 W 194	10 x 1	228	21.1	15,100	17.7	26,860	18.0	16.2	515
100.0	44.0	36 W 280	10 x 1/2	297	19.3	20,370	17.9	34,740	18.4	15.3	525

Stringer Spacing 8' 0", n = 10, t = 7", 12t = 84"

Span Foot	h"	Wide Flange Section	Bottom Cover Plate	wt. lbs.	Non Composite C_1"	Non Composite I" 4	Composite C_2"	Composite I" 4	f_{st} ksi	f_{sc} ksi	f_c psi
50.0	37.8	30 W 108	7 x 1	132	17.7	5820	12.0	16,280	18.2	12.7	460
62.5	41.1	33 W 130	10 x 1	164	20.5	9020	14.4	24,300	17.5	14.8	435
75.0	44.5	36 W 194	10 x 1	228	21.0	15,100	16.6	33,960	17.9	15.4	478
87.5	44.5	36 W 280	7 x 1	304	19.7	21,100	17.5	40,230	18.0	14.6	483
100.0	44.5	36 W 280	18 x 1	341	21.7	24,000	19.3	47,450	18.2	18.5	535

Table 5. Design Properties and Maximum Stresses for Composite Wide Flange Beams, Supported for Dead Load H20 S16 Live Load

Stringer Spacing 6' 0", n = 30 DL, n = 10 LL t = 6" 12t = 72"

Span Feet	h"	W Section	Bottom Cover Plate	wt. lbs.	n = 30 C_3"	n = 30 I"4	n = 10 C_2"	n = 10 I"4	f_{st} ksi	f_{sc} ksi	f_c psi
50	36.3	30 W 108	6 x 1/2	118	16.6	8900	10.7	12,320	17.4	5.6	724
50	35.8	30 W 108		108	15.4	7640	10.6	10,340	20.7	5.7	776
75	42.8	36 W 150	10 x 1	184	21.1	18,120	16.3	26,400	17.7	9.2	863
100	43.5	36 W 280	12 x 1	321	23.4	29,460	19.2	39,250	17.0	12.1	995

Stringer Spacing 8' 0", n = 30 DL, n = 10 LL t = 7" 12t = 84"

Span Feet	h"	W Section	Bottom Cover Plate	wt. lbs.	n = 30 C_3"	n = 30 I" 4	n = 10 C_2"	n = 10 I"4	f_{st} ksi	f_{sc} ksi	f_c psi
50	42.8	36 W 150		150	18.3	14,240	12.7	21,130	17.3	5.0	626
75	43.9	36 W 230	9 x 1	261	22.1	26,500	16.8	36,350	17.3	9.1	921
100	45.8	36 W 300	10 x 1 top 20 x 1 bottom	402	24.6	41,800	19.9	56,800	16.9	11.3	959
100	45.8	36 W 300	5 x 1 top 20 x 1 bottom	385	25.3	39,050	20.2	53,420	17.5	12.5	1040
100	44.8	36 W 300	20 x 1 bottom	368	24.2	36,800	19.3	49,600	18.7	12.8	1100

Table 6. Design Properties and Maximum Stresses for Composite Wide Flange Beams, Supported for Dead Load, H15 S12 Live Load

Stringer Spacing 6' 0" t = 6" , 12t = 72" n = 30 DL, n = 10 LL

Span Feet	h"	W Section	Bottom Cover Plate	wt. lbs.	n = 30 C_3	n = 30 $I"^4$	n = 10 C_2	n = 10 $I"^4$	f_{st} ksi	f_{sc} ksi	f_c psi
50.0	35.8	30 W 108		108	16.3	7930	10.6	10,320	17.4	5.1	615
75.0	40.1	33 W 130	10 x 1	164	20.8	13,500	14.8	21,100	17.5	9.7	836
100.0	43.5	36 W 280		280	21.1	24,360	17.0	31,600	19.3	11.4	910
100.0	44.0	36 W 280	10 x 1/2	297	22.1	26,570	17.9	34,740	17.4	11.6	893

Stringer Spacing 8' 0" t = 7", 12t = 84" n = 30 DL, n = 10 LL

Span Feet	h"	W Section	Bottom Cover Plate	wt. lbs.	n = 30 $C_3"$	n = 30 $I"^4$	n = 10 $C_2"$	n = 10 $I"^4$	f_{st} ksi	f_{sc} ksi	f_c psi
50.0	37.8	30 W 108	7 x 1	132	17.7	11,700	12.0	16,280	16.3	5.2	639
	36.8	30 W 108		108	14.9	8,571	10.0	11,460	22.7	5.1	722
75.0	44.5	36 W 194	10 x 1	228	20.7	24,370	16.6	33,960	16.8	7.9	735
100.0	44.5	36 W 280	18 x 1	341	25.3	34,450	19.3	47,450	16.6	12.4	950
100.0	44.5	36 W 280	15 x 1	331	24.1	33,170	18.9	46,970	17.7	12.1	941

Table 7. Design Properties and Maximum Stresses for Composite Welded Plate Girders Unsupported for Dead Load H20 S16 Live Load

Stringer Spacing 6' 0" t = 6" 12t = 72"

Span Feet	h"	Web Plate	Top Plate	Bottom Plate	wt. lbs.*	Non Composite C_1	Non Composite $I"^4$	Composite C_2	Composite $I"^4$	f_{st} ksi	f_{sc} ksi	f_c psi
50	43.5	36 x 3/8	4 x 1/2	10 x 1	96	24.8	4760	13.3	17,200	14.7	17.0	456
62.5	49.5	42 x 3/8	5 x 1/2	12 x 1	112	28.8	7630	16.2	25,660	17.6	18.0	596
75	56.0	48 x 3/8	4 x 1	14 x 1	133	31.8	12,480	18.8	36,430	17.7	17.5	500
87.5	56.0	48 x 3/8	6-1/2 x 1	19 x 1	158	32.0	16,600	17.5	44,060	18.6	18.4	453
100	68.0	60 x 3/8	7 x 1	18 x 1	172	38.0	28,000	24.5	66,160	18.1	18.5	487

Stringer Spacing 8' 0" t = 7" 12t = 84"

Span Feet	h"	Web Plate	Top Plate	Bottom Plate	wt. lbs.*	Non Composite C_1	Non Composite $I"^4$	Composite C_2	Composite $I"^4$	f_{st} ksi	f_{sc} ksi	f_c psi
50	50.5	42 x 3/8	4 x 1/2	11 x 1	110	28.7	7020	14.3	27,000	17.8	15.6	413
62.5	56.0	48 x 3/8	4 x 1	15 x 1	136	32.3	12,900	17.4	42,000	17.4	16.5	435
75	56.0	48 x 3/8	7 x 1	20-1/2 x 1	165	32.3	17,840	19.1	50,740	17.7	18.0	490
87.5	63.0	54 x 3/8	9 x 1	23 x 1	188	35.4	26,300	21.3	68,240	18.5	18.4	477
100	69.25	60 x 3/8	10-1/2x1	20 x 1-1/4	207	38.6	37,400	24.5	89,200	18.0	18.5	476

* Includes Stiffener Weight

Table 8. Design Properties and Maximum Stresses for Composite Welded Plate Girders Unsupported for Dead Load, H15 S12 Live Load

Stringer Spacing 6' 0", n = 10, t = 6" , 12t = 72"

Span Feet	h"	Web Plate	Top Plate	Bottom Plate	wt.* lbs.	Non C_1"	Composite I"4	C_2"	Composite I"4	f_{st} ski	f_{sc} ksi	f_c psi
50	43.5	36 x 3/8	3 x 1/2	7-1/2 x 1	86	23.9	3990	12.3	14,750	17.7	15.8	361
62.5	49.5	42 x 3/8	5 x 1/2	9-1/2 x 1	103	27.4	7040	14.9	22,700	17.7	17.2	385
75	50	42 x 3/8	5 x 1	14 x 1	127	27.6	10,000	16.6	28,000	17.8	17.9	441
87.5	56	48 x 3/8	6-1/2 x 1	16 x 1	148	30.7	15,840	19.3	39,600	17.8	18.0	406
100	62	54 x 3/8	8 x 1	18 x 1	167	34.0	22,900	21.1	53,600	17.5	17.6	382

Class of Live Load H15 S12 Stringer Spacing 8' 0" t = 7" 12t = 84"

Span Feet	h"	Web Plate	Top Plate	Bottom Plate	wt.* lbs.	Non C_1"	Composite I"4	C_2"	Composite I"4	f_{st} ksi	f_{sc} ksi	f_c psi
50	445	36 x 3/8	6 x 1/2	11 x 1	102	24.4	5450	12.4	20,000	17.9	16.2	368
62.5	51	42 x 3/8	5 x 1	14 x 1	127	27.6	10,250	15.1	31,400	17.6	17.1	373
75	51	42 x 3/8	8-1/2 x 1	20 x 1	159	26.7	14,140	16.4	43,150	18.5	17.8	428
87.5	57	48 x 3/8	10 x 1	22 x 1	180	30.9	21,450	19.1	53,450	17.7	18.2	395
100	69	60 x 3/8	10 x 1	22 x 1	198	37.7	34,060	23.1	82,660	17.8	18.5	373

* Includes Stiffener Weight

Table 9. Design Properties and Maximum Stresses for Composite Welded Plate Girders Supported for Dead Load H20 S16 Live Load

Class of Live Load H20 S16 Stringer Spacing 6' 0", n = 30 DL, n = 10 LL, t = 6", 12t = 72"

Span Feet	h"	Web Plate	Top Plate	Bottom Plate	wt.* lbs./ft.	n = 30 C_3"	n = 30 I" 4	n = 10 C_2"	n = 10 I" 4	f_{st} ksi	f_{sc} ksi	f_c psi
50	43.5	36 x 3/8	4 x 1/2	8 x 1	89	19.6	11,100	12.4	15,600	18.2	5.3	601
75	56	48 x 3/8	3 x 1	13 x 1	126	27.8	24,560	18.5	35,660	17.1	8.3	810
100	68	60 x 3/8	5 x 1	16 x 1	158	34.4	42,500	24.0	61,460	18.1	11.2	812

Stringer Spacing 8' 0", n = 30 DL, n = 10 LL, t = 7" 12t = 84"

Span Feet	h"	Web Plate	Top Plate	Bottom Plate	wt.* lbs./ft.	n = 30 C_3"	n = 30 I" 4	n = 10 C_2"	n = 10 I" 4	f_{st} ksi	f_{sc} ksi	f_c psi
50	50.5	42 x 3/8	4 x 1/2	8 x 1	97	20.7	16,510	12.7	76,600	17.4	45	509
75	56.0	48 x 3/8	4 x 1	18 x 1	146	28.3	30,520	18.5	45,070	18.5	9.1	786
100	69.2	60 x 3/8	5 x 1	20 x 1-1/4	188	37.2	58,650	25.3	87,450	17.0	11.3	820
100	69.0	60 x 3/8	5 x 1	22 x 1	179	35.7	53,670	24.0	78,680	19.2	11.7	859

* Includes Stringer Weight

Table 10. Design Properties and Maximum Stresses for Composite Welded Plate Girders Supported for Dead Load H15 S12 Live Load

Class of Live Load H15 S12 Stringer Spacing 6' 0", n = 30 DL, n = 10 LL , t = 6" 12t = 72"

Span Feet	h"	Web Plate	Top Plate	Bottom Plate	wt.* lbs/ft.	n = 30 C_3"	n = 30 I" 4	n = 10 C_2"	n = 10 I" 4	f_{st} ksi	f_{sc} ksi	f_c psi
50.0	43.5	36 x 3/8	3 x 1/2	5-1/2 x 1		18.0	9240	11.2	12,800	17.8	4.8	329
75.0	49.5	42 x 3/8	3 x 1	12 x 1		24.1	17,650	15.9	26,710	17.5	8.4	522
100.0	62	54 x 3/8	6 x 1	15 x 1		30.3	33,860	21.0	48,040	18.6	10.7	735

Stringer Spacing 8' 0", n = 30 DL, n = 10 LL, t = 7" 12t = 84"

Span Feet	h"	Web Plate	Top Plate	Bottom Plate	wt.* lbs/ft.	n = 30 C_3"	n = 30 I" 4	n = 10 C_2"	n = 10 I" 4	f_{st} ksi	f_{sc} ksi	f_c psi
50	44.5	36 x 3/8	6 x 1/2	11 x 1	102	19.6	14,370	12.4	20,000	15.3	4.6	410
50	44.5	36 x 3/8	6 x 1/2	8 x 1	92	18.2	12,000	12.2	16,500	19.0	5.0	352
75	51	42 x 3/8	6-1/2 x 1	18 x 1	145	24.7	25,350	16.3	36,000	17.7	9.1	616
100	69	60 x 3/8	5 x 1	19 x 1	171	34.2	51,450	22.9	74,650	18.0	10.6	609

* Includes Stiffener Weight

Table 11

Weight of Stiffener Per Foot of Span at Maximum Stiffener Spacing	
Depth of Web	Weight per Foot
36"	8.9#
42"	8.9
48"	10.2
54"	10.2
60"	10.2

FIGURES

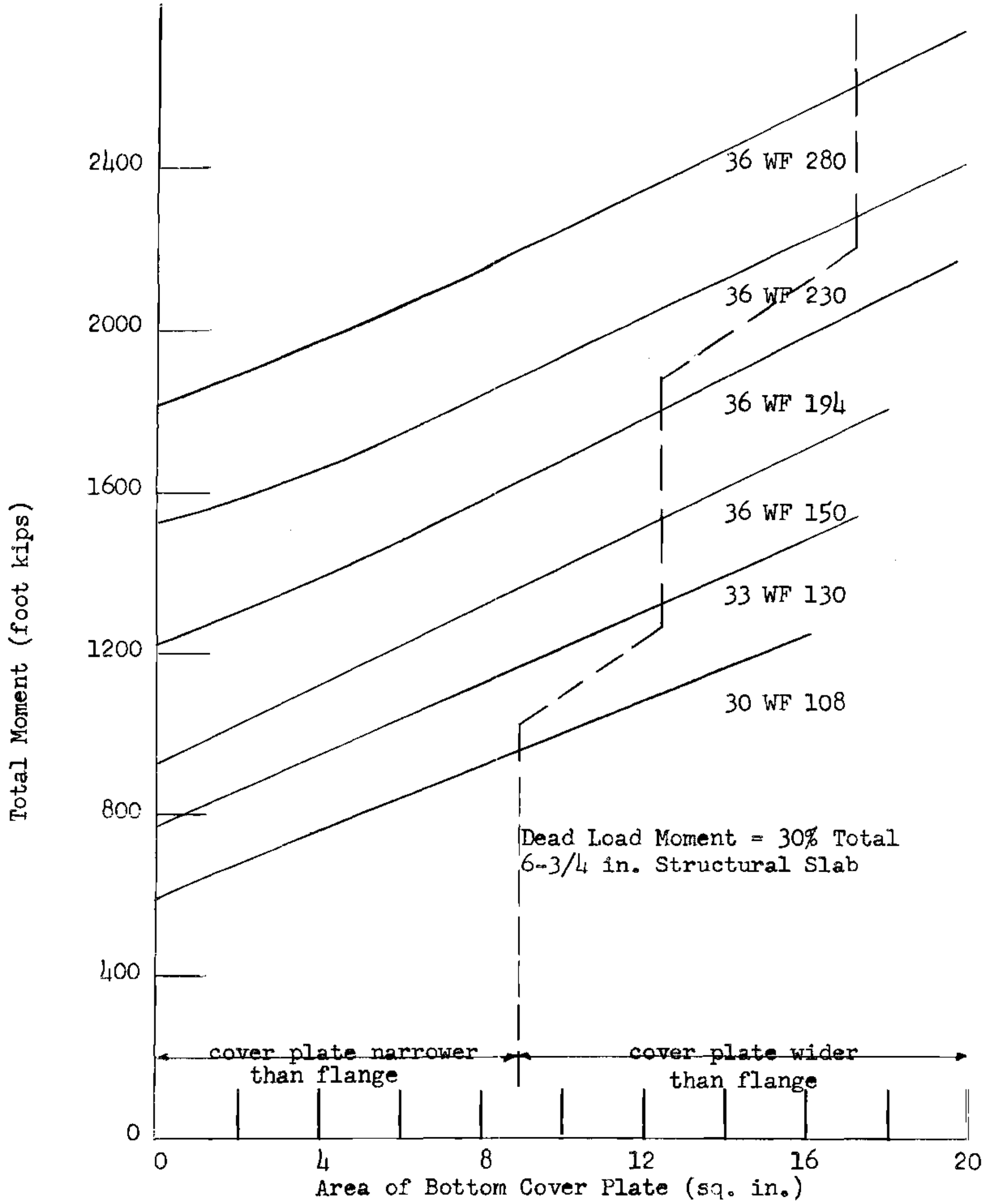


Fig. 5. Graph for Selection of Composite Wide Flange Beams

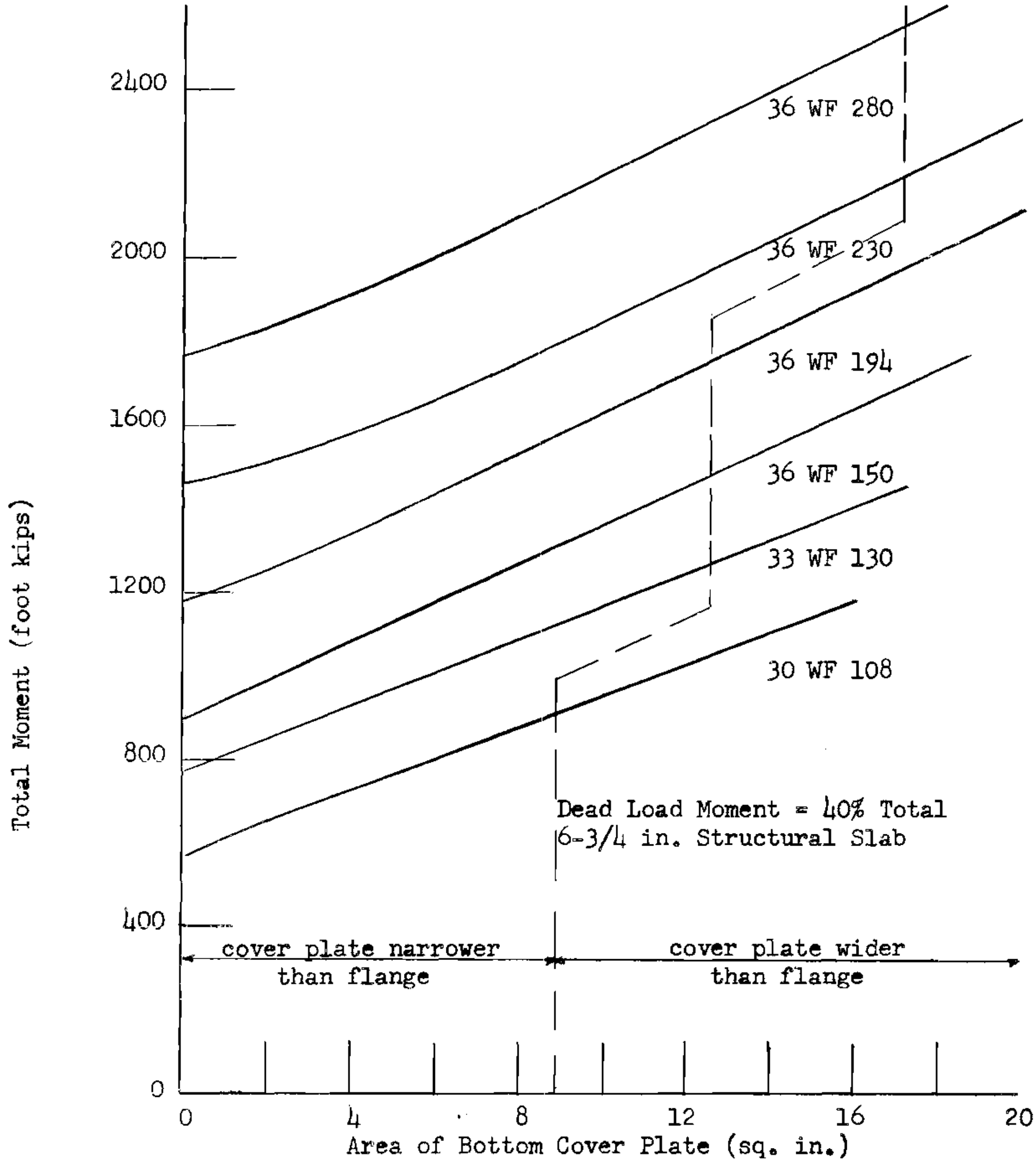


Fig. 6. Graph for Selection of Composite Wide Flange Beams

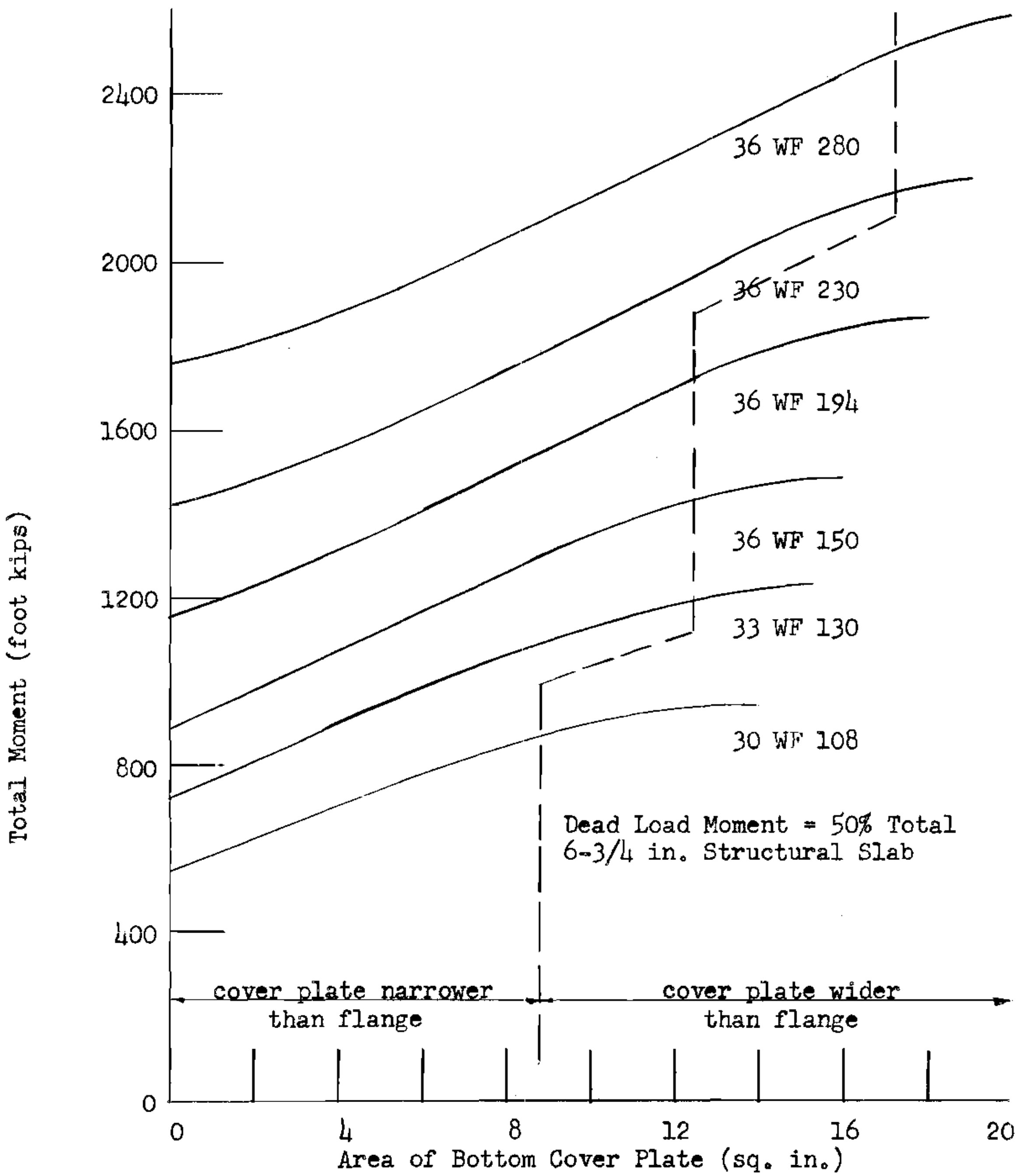


Fig. 7. Graph for Selection of Composite Wide Flange Beams

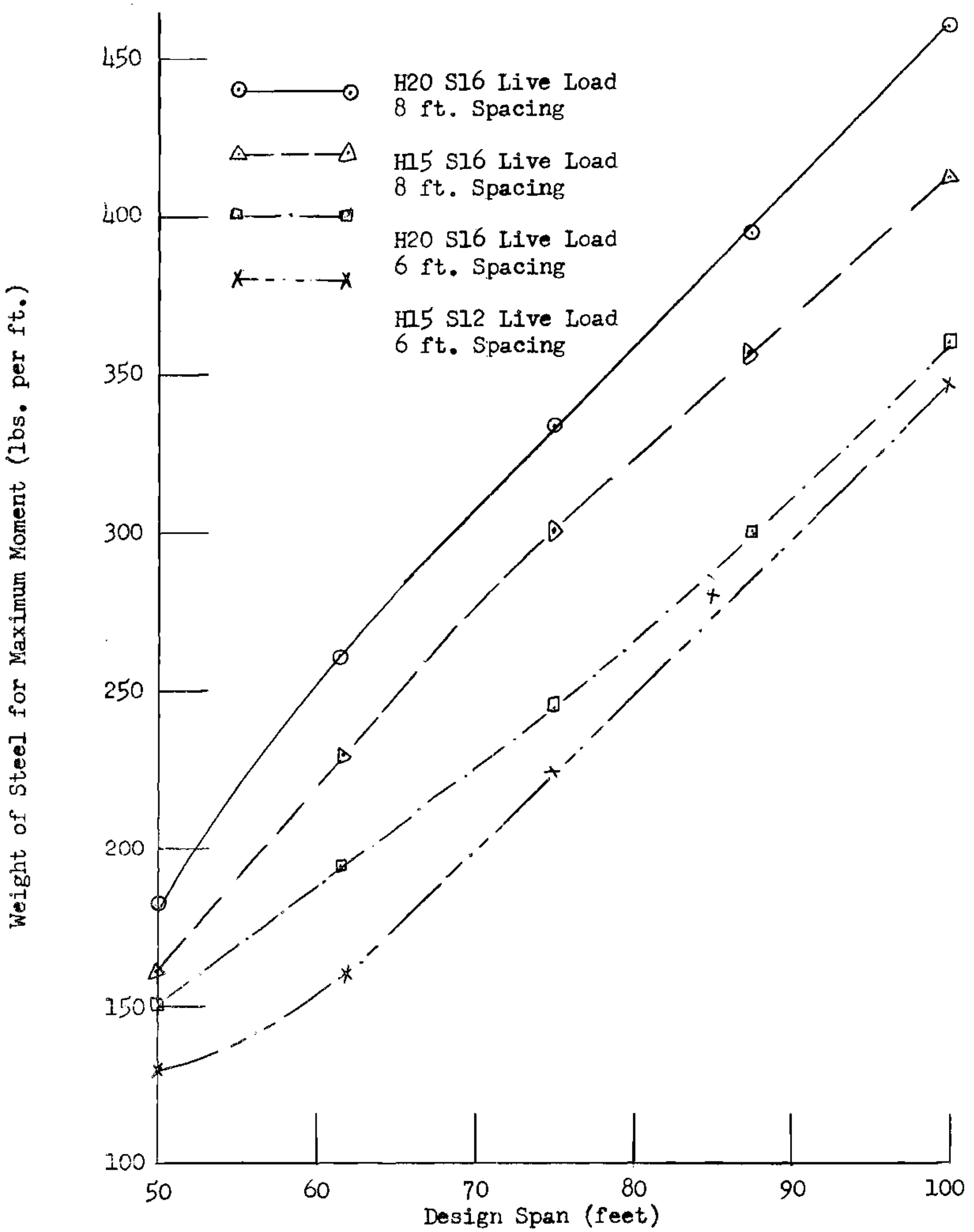


Fig. 8. Weight of Non Composite Wide Flange Beams Compared to Span Length

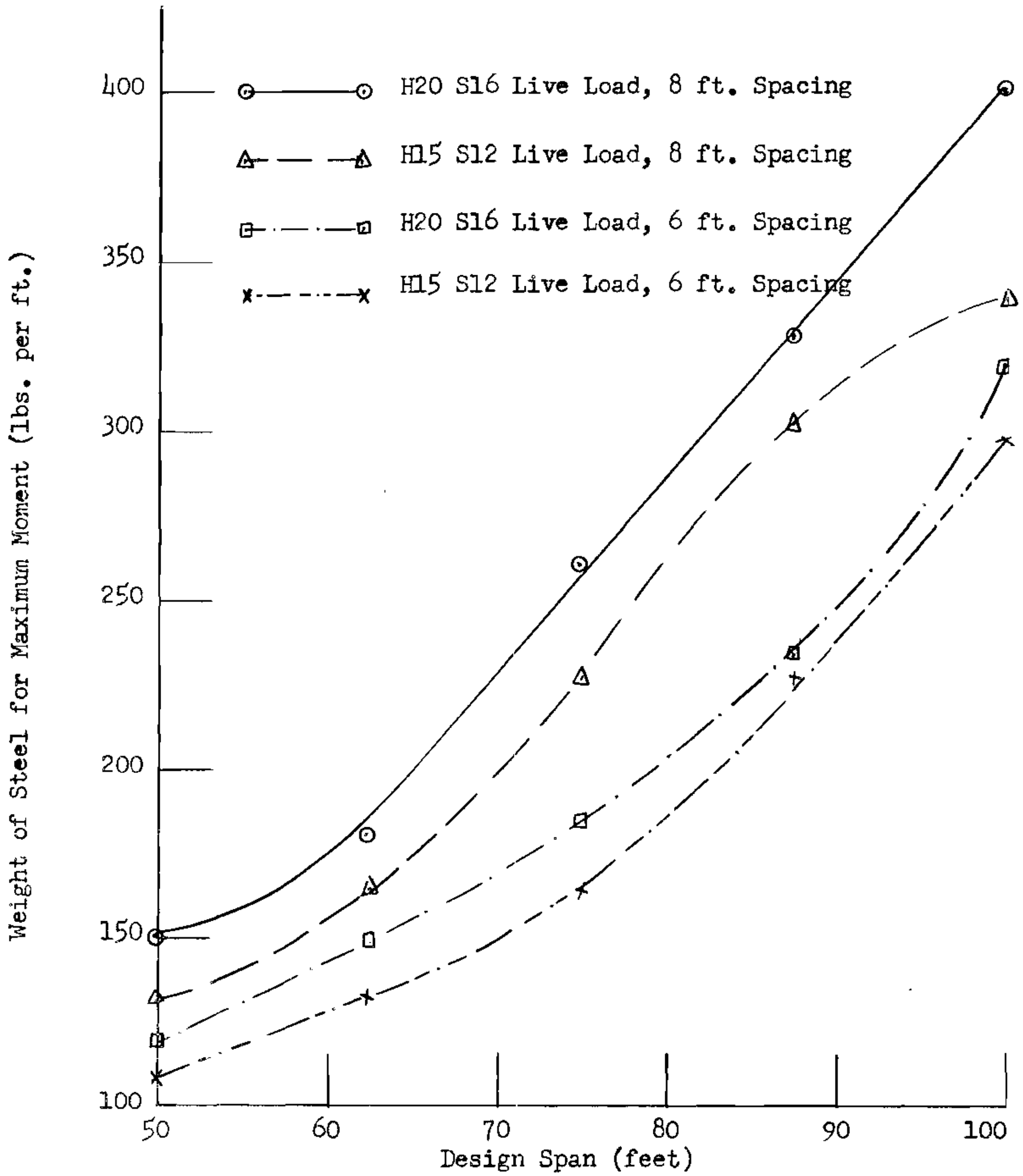


Fig. 9. Weight of Composite Wide Flange Beams
Compared to Span Length

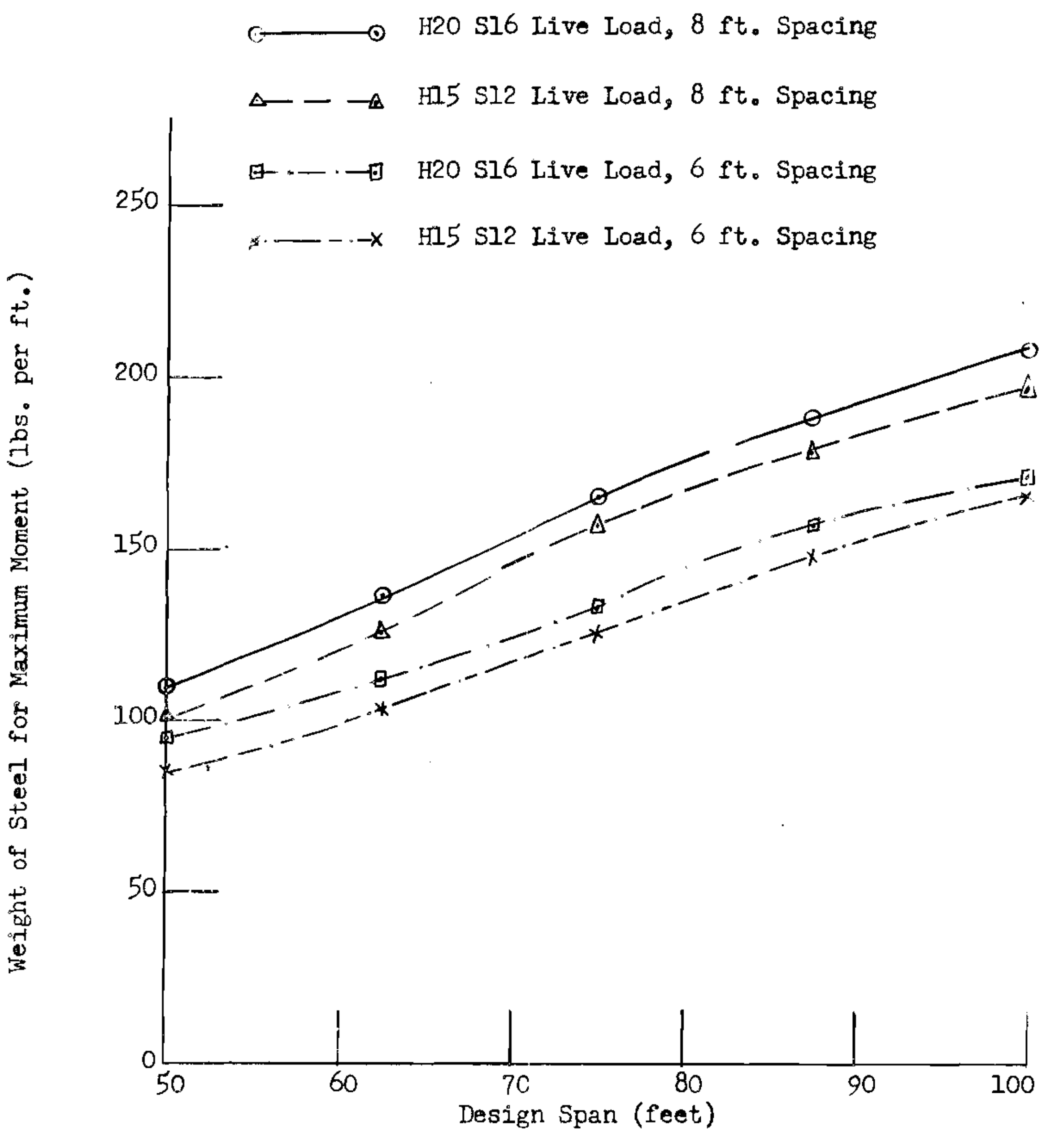


Fig. 10. Weight of Composite Welded Plate Girders Compared to Span Length

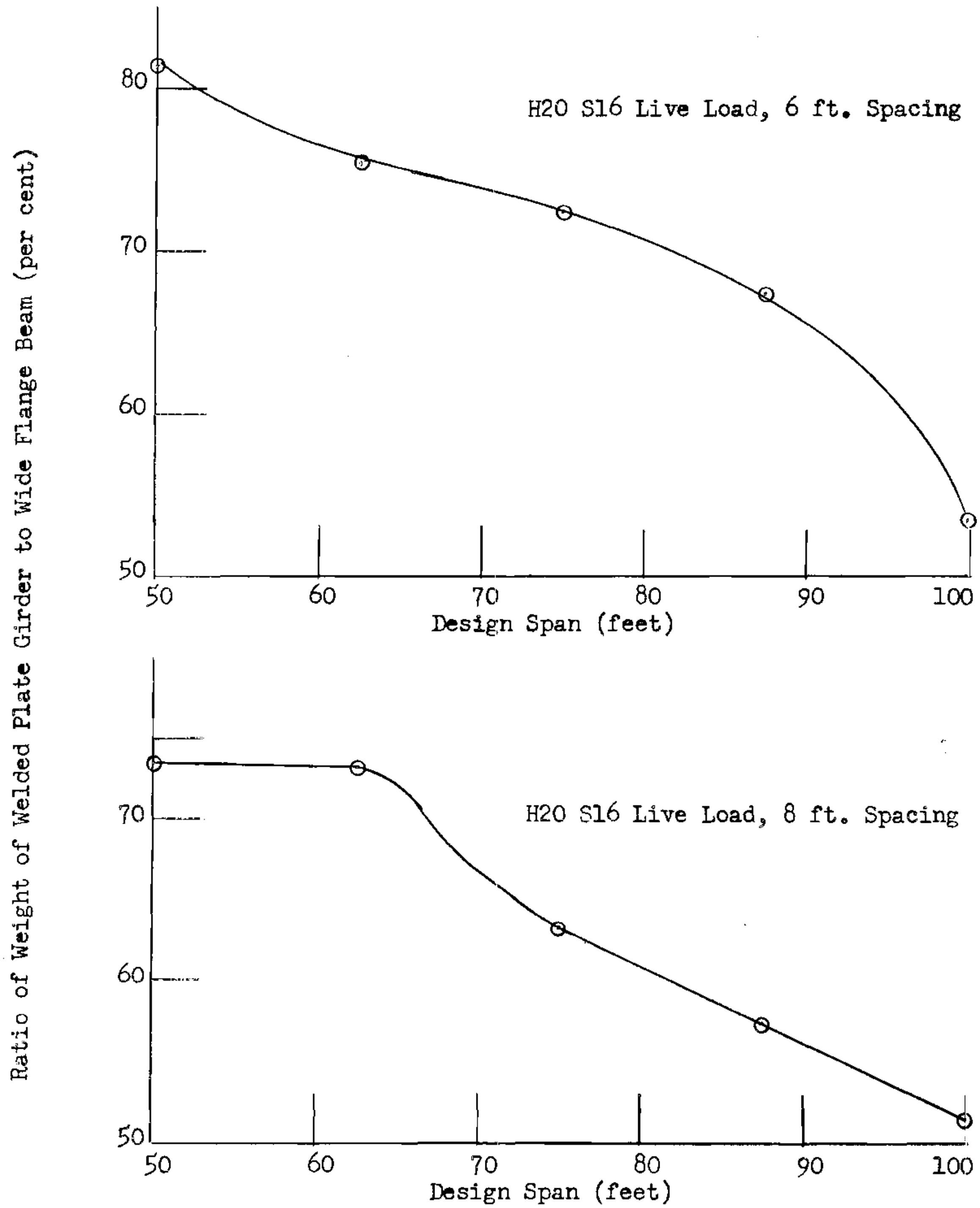


Fig. 11. Ratio of Steel Weight of Composite Stringers Compared to Span Length

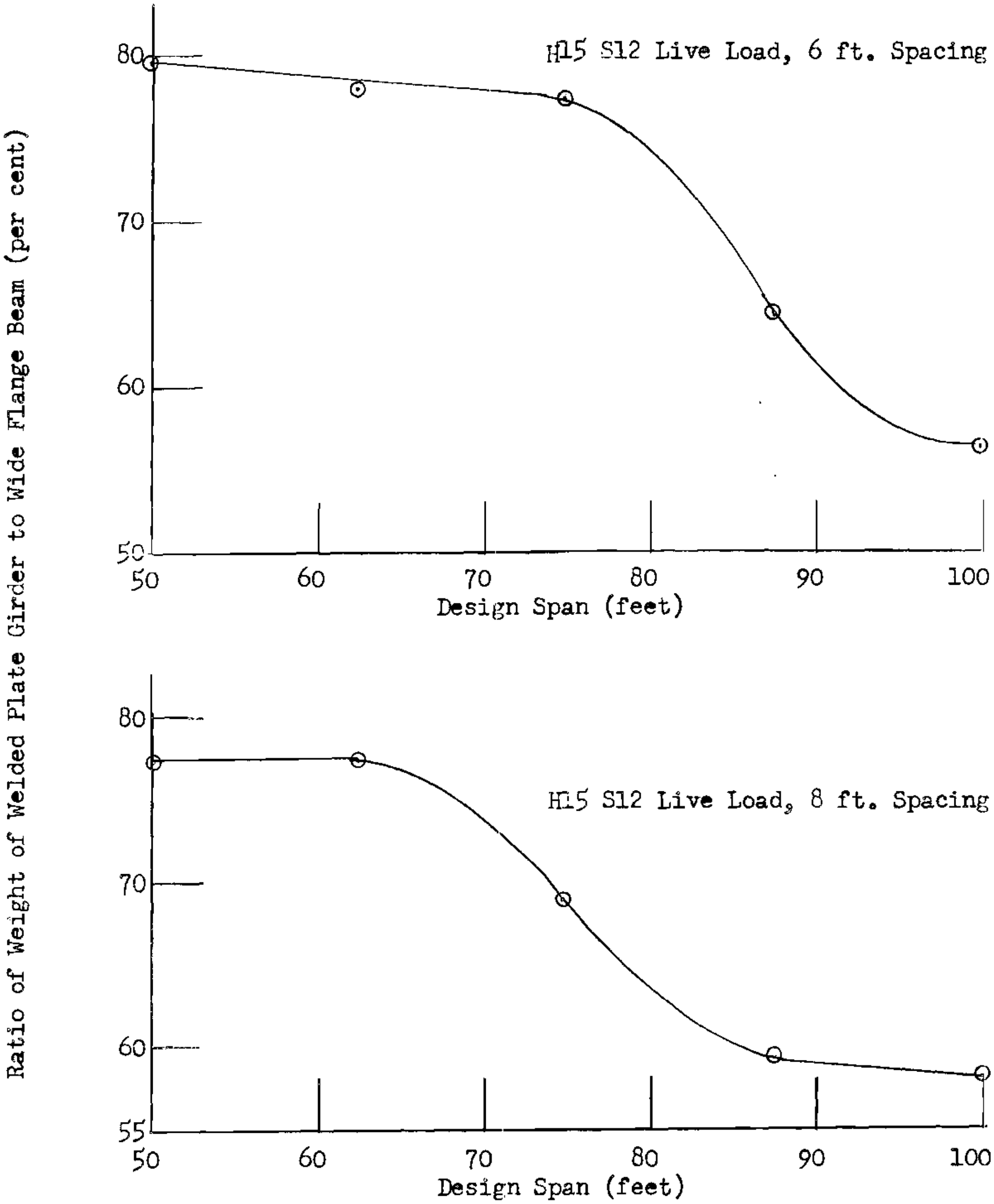


Fig. 12. Ratio of Steel Weight of Composite Stringers
Compared to Span Length

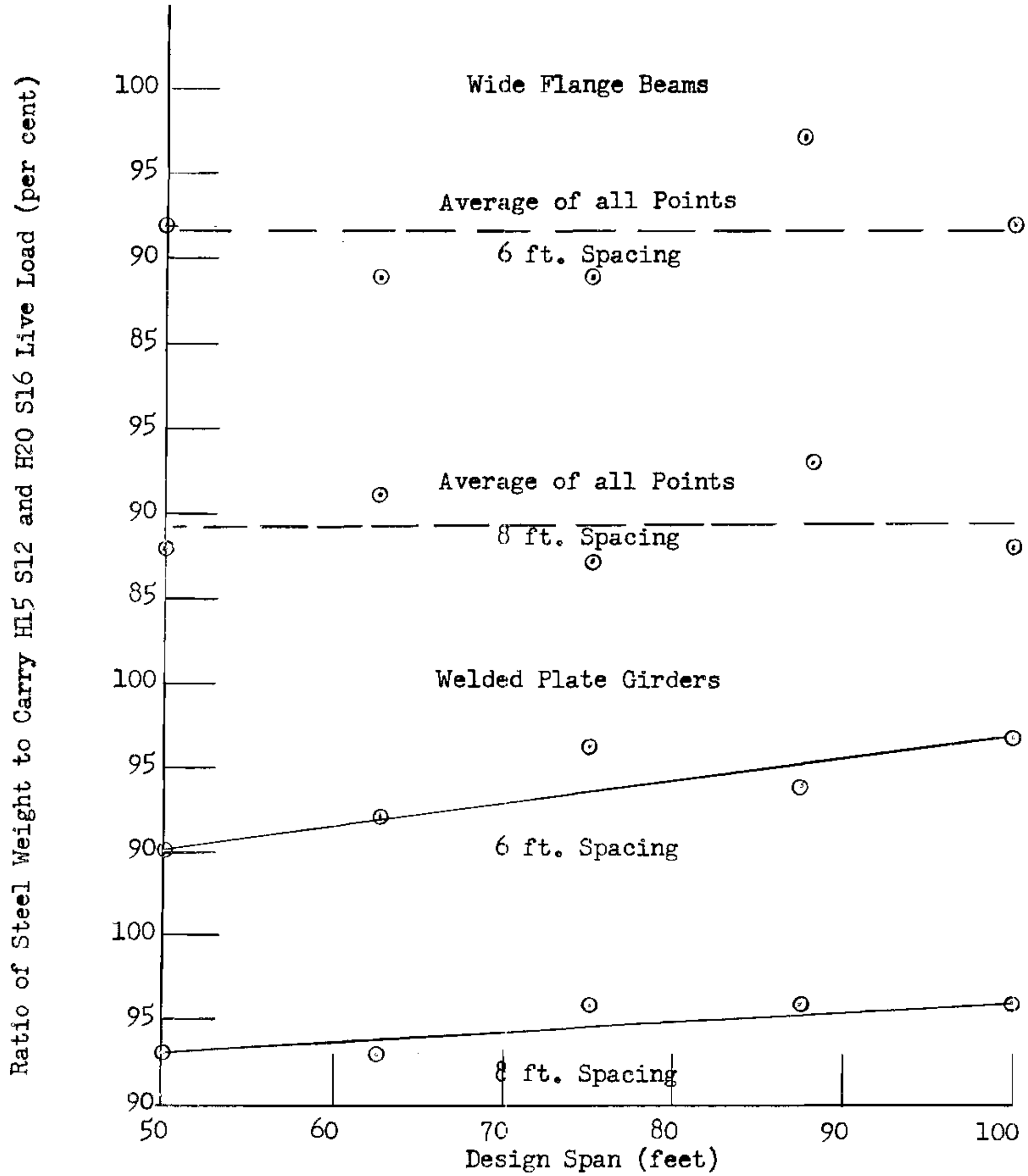


Fig. 13. Ratio of Steel Weight to Carry Live Load
Compared to Span Length

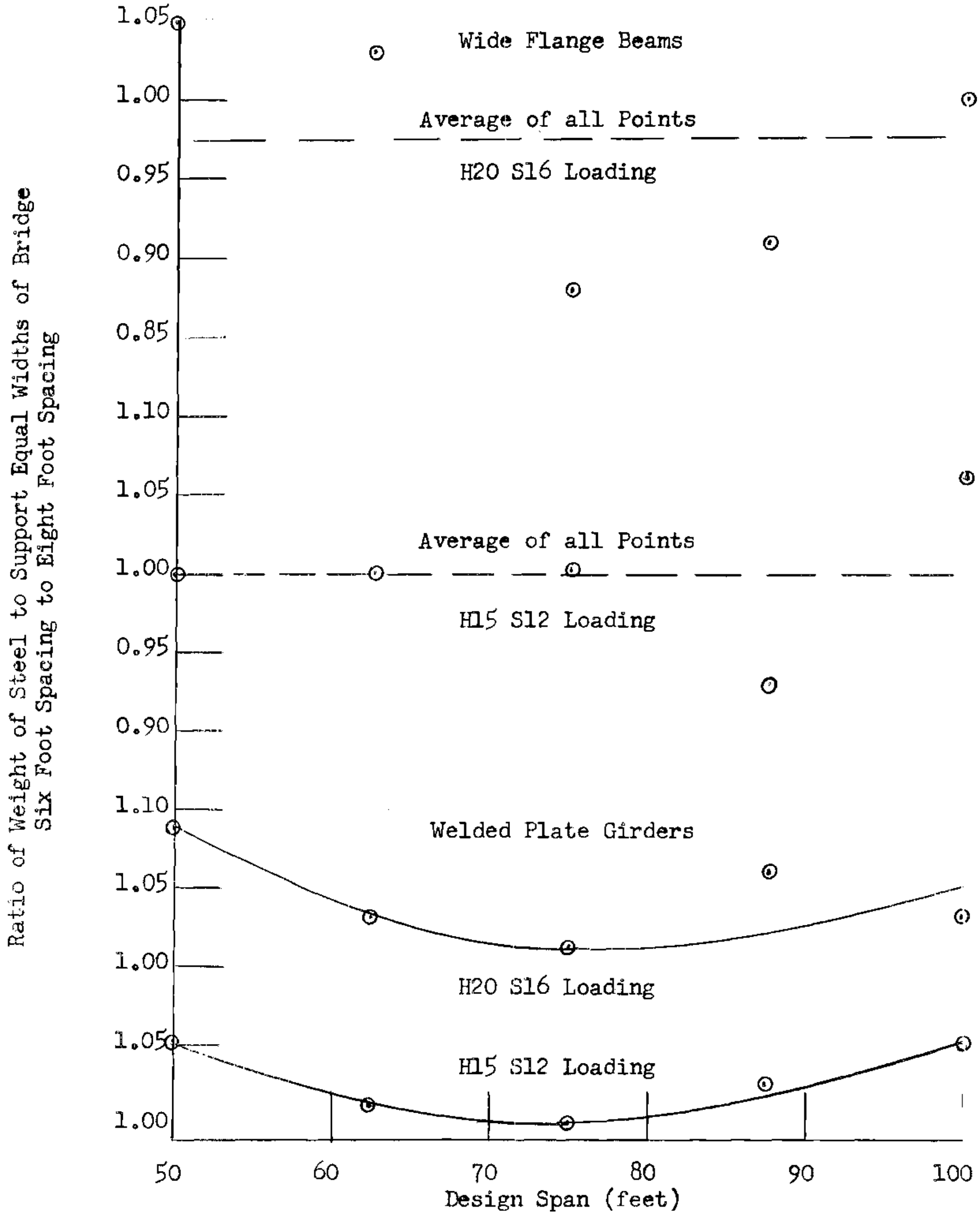


Fig. 14. Ratio of Weight of Steel to Support Equal Widths of Bridge Compared to Span Lengths

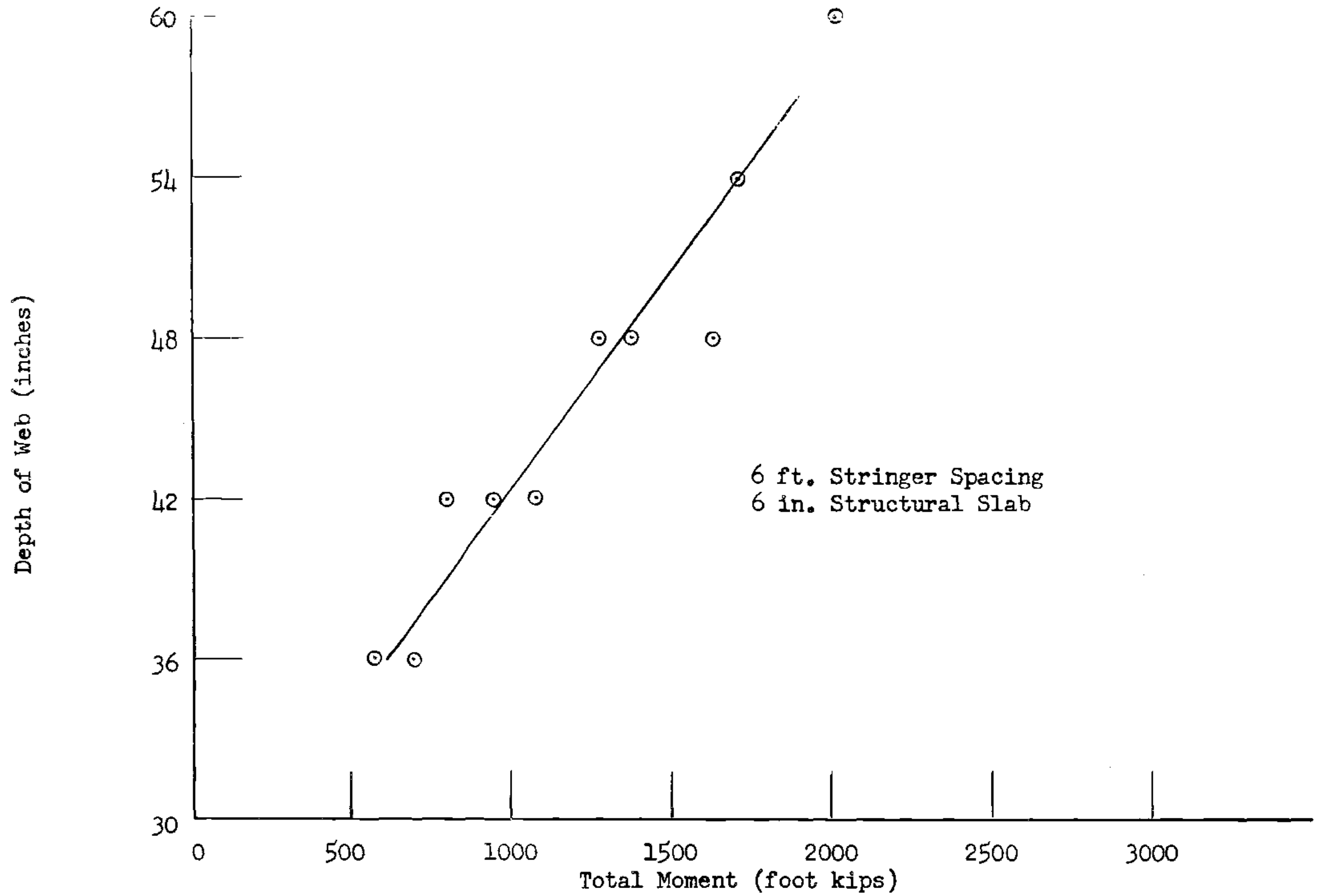


Fig. 15. Depth of Web Required for Total Moment Composite Welded Girders

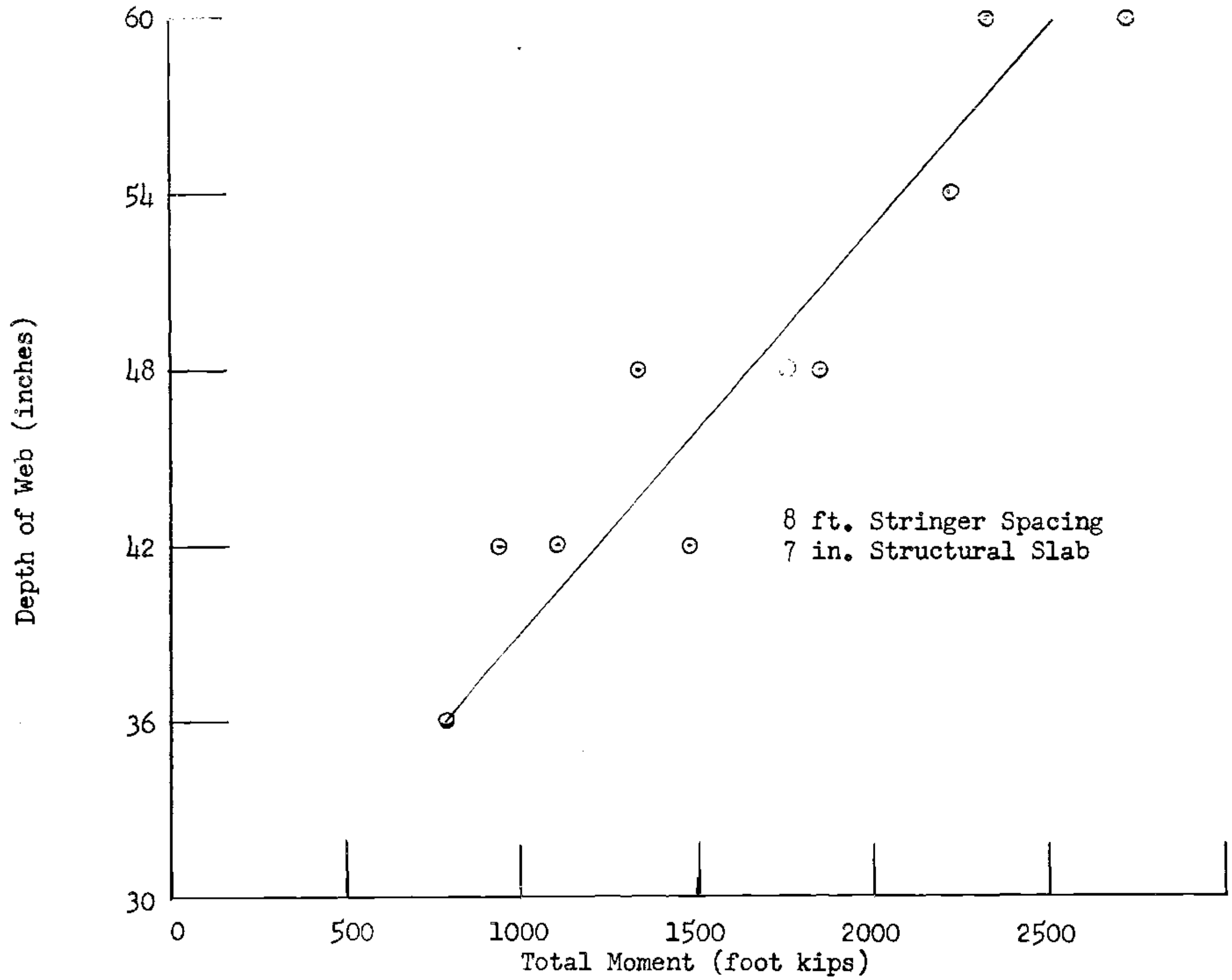


Fig. 16. Depth of Web Required for Total Moment Composite Welded Girders

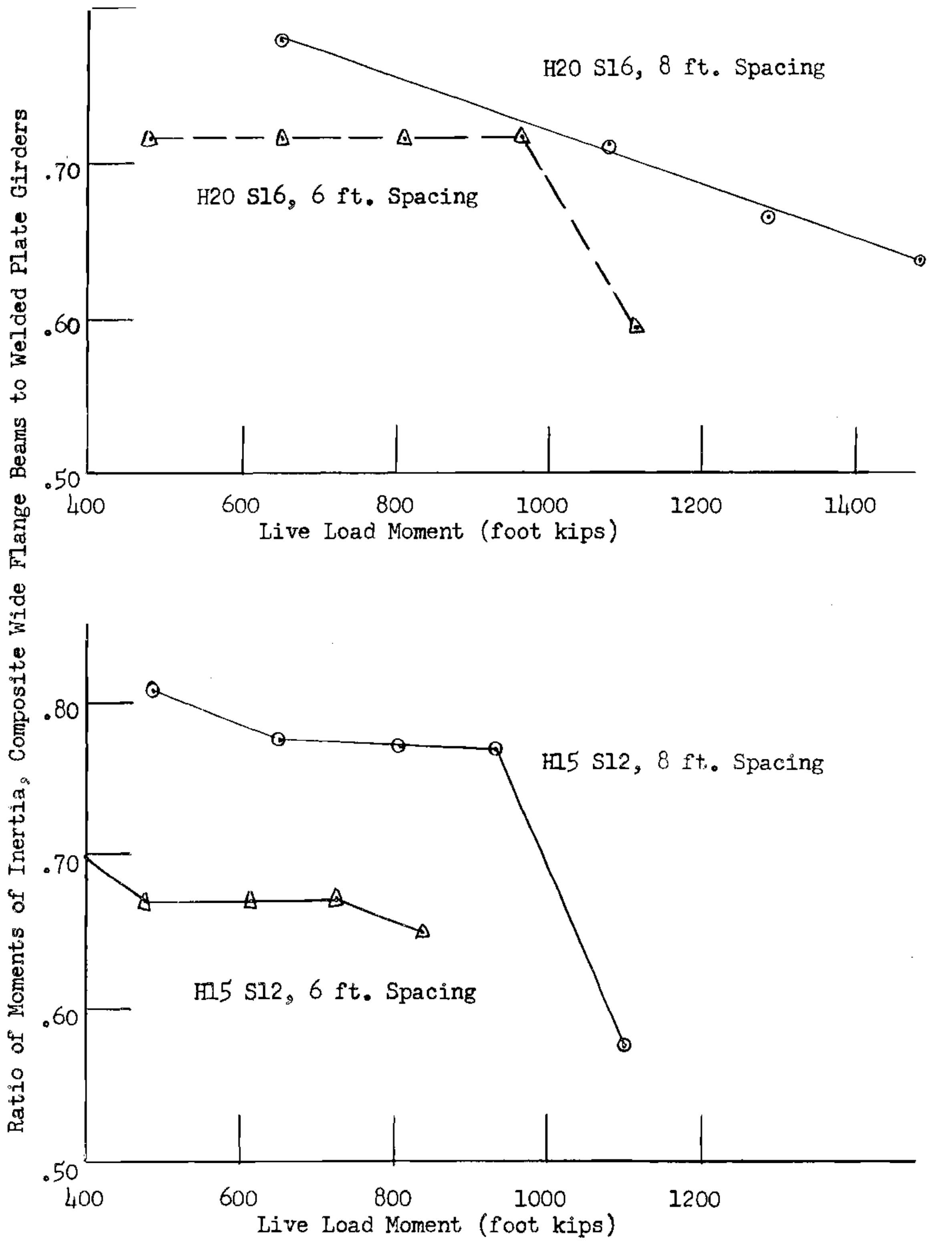


Fig. 17. Ratio of Moments of Inertia Compared to Live Load Moment

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